Seismic Behavior Factors of Steel Frames Braced with Viscoelastic Damping System

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Abstract. In this study a number of seismic behavior factors (overall ductility, response modification, and overstrength) of ordinary moment steel frames with viscoelastic bracing system were evaluated. These factors are not provided for ordinary moment steel frames with viscoelastic bracing system in building seismic codes such as the International Building Code (IBC) or Euro Code (EN). Moreover, similar frames without viscoelastic bracing were assessed and compared as well. A linear history analysis both two types with a different number of stories and span lengths was carried out using different earthquake records, which were selected to include variability in ground motion characteristics. Pushover analysis was then performed after defining the sizes of the elements and assigning material nonlinearity to the discrete hinge where plastic rotation occurs to beams and columns according to FEMA 356. Such analysis allows evaluating the overall ductility and the overstrength of each building of concern by using the yield and ultimate displacements and base shear forces obtained from the pushover curve. The results showed that overall ductility, overstrength, and response modification decreased with an increase of the number of stories for all buildings or when the bay length increased. Adding viscoelastic dampers increased the seismic behavior factors for all buildings significantly.

Keywords: *overall ductility factor; overstrength factor; response modification factor; steel frames; viscoelastic bracing.*

1 Introduction

Based on statistics of the frequency and magnitude of earthquakes, more than 200 large magnitude earthquakes occur each decade [1]. This natural action can result in serious damage to human life, economy, and structures in general [2]. The number of people that are lost or killed due to earthquakes has dramatically increased over time [3]. Life safety has historically been a major concern in earthquake design to prevent human casualties [4]. However, based on the statistics, more than three-quarters of the cities in the world with more than 10

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million people are situated in areas that can be affected by a serious earthquake at any time throughout the year [5].

In general, strength is mainly associated with structural damage control [6]. On the other hand, ductility, low weight, and flexibility are considered to be essential parameters in every structure to resist earthquakes. Steel structures can efficiently resist earthquakes since they have such properties [7]. Deformation under seismic force will increase significantly only if the structure loses elasticity, which will lead the stiffness to drop in a serious way [8]. Accordingly, every structure should match the requirements of remaining stable without collapsing when deformation increases, or in other words, to retain their vertical load carrying capacity [9].

The resistance of any structure towards lateral movement without collapsing is known as ductility. Ductility is defined as the ratio of ultimate strain to yield strain of the material. Earthquake engineering shows an obvious focus on understanding the ductility term as well as the force reduction term to provide the knowledge needed to realize the required ductility capacity to meet ductility demand when designing cost-effective structures that can survive earthquake excitations [10]. Ductility detailing is only needed when the design does not match the requirements of being elastic under serious levels of earthquake vibrations [11]. Feasibility in engineering is just as important as the economic part. Establishing an elastic structure is costly and difficult. However, as long as the structure has a vertical load carrying capacity under high levels of load that can cause deformation, it can be taken as an extra option that can be applied [12]. Structures that can sustain and resist earthquake excitations are designed based on ultimate strength less than elastic strength demand by a ratio of two to eight, as long as the structure has properties such as frequency shift, ductility, and energy dissipation capacity [9].

The stiffness in any structural component will only drop once the material has become more compliant. However, the internal forces along with the total base shear that represents the total of the internal shear forces in the entire vertical load-carrying element will be considered higher compared to the external forces on the structure when the structure is collapsed. In that case, the entire vertical load-carrying element remains elastic [13]. The seismic force (elastic force demand) can possibly be reduced to the design level (inelastic force demand) through the application of the seismic response modification factor, *R*, provided that the structure has adequate strength and its elements are ductile enough. This concept has been adopted by several seismic design codes in the analysis and design of earthquake resistant structures [14]. Systems are enabled to undergo low values of deformation before collapsing by the ductility detailing [15].

Viscoelastic damping systems have been adopted for several tall buildings all over the world to reduce seismic effects. Major reduction of lateral movement by such systems is gained in those buildings. Recent researches have been conducted and have proved that viscoelastic damping is suitable for seismic resistant structures [16-21]. The seismic factors of overall ductility, response modification, and overstrength for ordinary moment steel frames equipped with viscoelastic damping system are not given in building codes such as IBC. The present study included an assessment of these factors for ordinary moment steel frames with and without a viscoelastic damping system. Bay length and number of stories were varied as factors that influence the response of the building. Buildings with three, six, nine, twelve, and fifteen stories were considered as well as span lengths of five, seven, and nine meter.

2 Overall Ductility Factor μ

Ductility is usually defined as the capacity of a structure to sustain large inelastic deformations without collapse and any major reduction in strength and stiffness [9]. For economic reasons, most structures are designed and constructed to behave plastically under severe earthquakes. The response to earthquake-induced vibrations depends on the energy dissipation level of the structure, which is a function of its capability to absorb and dissipate energy by ductile deformation [9]. The overall ductility factor is the most common indicator of seismic designs and is defined as the ratio between ultimate displacement and yield displacement, and can be represented as follows, where Δ_u and Δ_y are displacements at the ultimate and the yield point, respectively (see Eq. (1),

$$\mu_{\text{overall}} = \frac{\Delta_u}{\Delta_y} \tag{1}$$

For multi-story buildings, the maximum and yield displacements that determine story ductility are measured at roof level to evaluate the overall ductility factor [3]. The overall system ductility, μ, can be defined as the weighted average of the story ductility factors and is calculated by considering a particular pattern of displacement corresponding to the fundamental mode shape or any other combination of mode shapes [22]. The effectiveness of the design approach involving the strong column-weak beam concept is still a controversial matter; it is dangerous to design a structure without taking into account the formation of plastic hinges in the columns [23]. Also, nonlinear deformation and formation of plastic zones are most likely to occur in the lower stories, while the walls of the upper stories will behave in the elastic range for multi-story frames [24].

3 Overstrength Factor, Ω

The overstrength factor is a parameter in seismic design and is defined as the ratio between maximum base shear and design base shear [3,21]. It can be represented by following Eq. (2):

$$\Omega = \frac{V_m}{V_y} \tag{2}$$

where V_m and V_y are maximum and design (or yield) base shear forces at the ultimate and the yield point, respectively. The overstrength parameter can reduce the elastic strength demand of a structure as well as maintain structural safety [21].

4 Response Modification Factor *R*

The *R* factor is the structural capacity required to maintain elasticity of a structure in the force-based seismic design method [21]. The *R* factor depends on the ductility, μ, and the overstrength, Ω [25]. Consequently, the *R* factor is determined in Eq. (3) [25] as follows,

$$R = R_\mu . \Omega \tag{3}$$

where R_μ is the ductility reduction factor and Ω is the overstrength factor.

R_μ is related to μ using the following equations, where T_n is the fundamental natural period of the structure (See Eqs. (4) to (6)) [22],

$$R_\mu = 1.0 \quad \text{for } T_n < 0.125 \text{ s} \tag{4}$$

$$R_\mu = \sqrt{2\mu\text{-}1} \quad \text{for } 0.125 \text{ s} < T_n < 0.5 \text{ s} \tag{5}$$

$$R_\mu = \mu \quad \text{for } T_n > 0.5\text{s} \tag{6}$$

5 Case Study and Analysis Methodology

5.1 Case Study and Building Description

Most constructions have an ordinary moment steel frame, whether they include a viscoelastic damping system or not. They should contain both orthogonal directions in the same floor plan. In this case study two types of lateral force resisting building systems were used – ordinary moment steel frames with and without viscoelastic damping systems – in order to calculate μ, Ω, and *R*. The steel frame constructions for the study were three, six, nine, twelve, and fifteen stories high. Also, the effect of different bay lengths on the seismic behavior factors were included in the study of a three-story steel frame building with span length varied at five, seven, and nine meter.

They had three bays with 5.0 m spacing in each horizontal and transverse direction for all stories of the building. All stories had a height of 3.0 m each. The applied design live load was 2.0 kN/m^2 on all floors, while the applied design dead load was 5.8 kN/m^2 on all floors. All columns and main beams had a steel section in the shape of an H-section of different sizes based on the linear time history analysis and the different number of stories. All secondary beams had a steel I-section of different sizes based on the linear time history analysis and the different number of stories. All steel elements were designed according to AISC360-10 LRFD provisions [26]. The unit weight of steel as applied was 76.8 kN/m^3. All steel elements were made of grade-50 steel with a yield and ultimate strength of 345 MPa and 448 MPa respectively. The stress-strain diagram definition of steel in ETABS is shown in Figure 1. Figure 2 shows the elevations of the ordinary moment steel frame system without viscoelastic dampers of a three-story building.

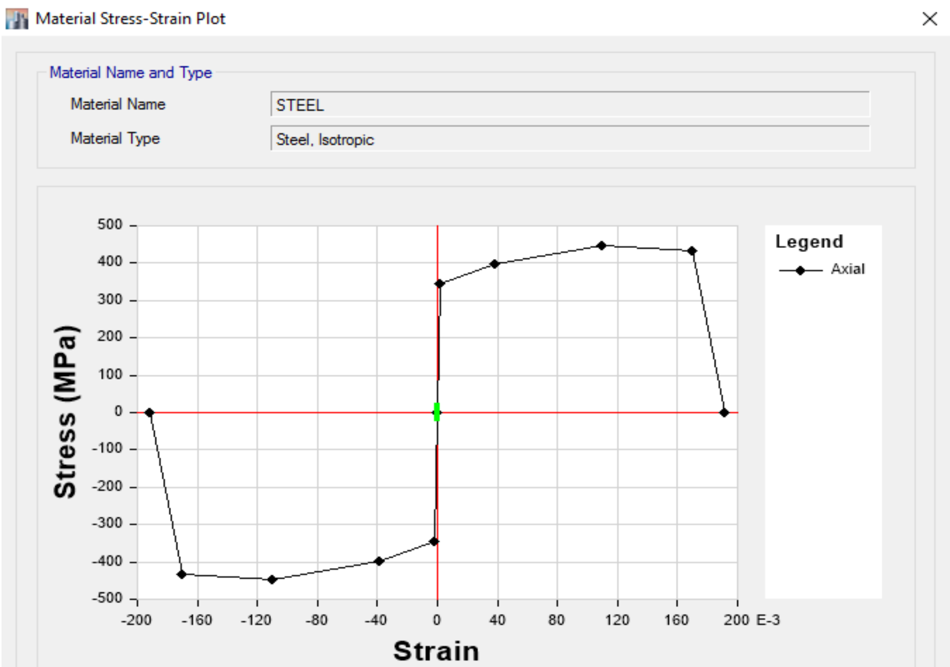
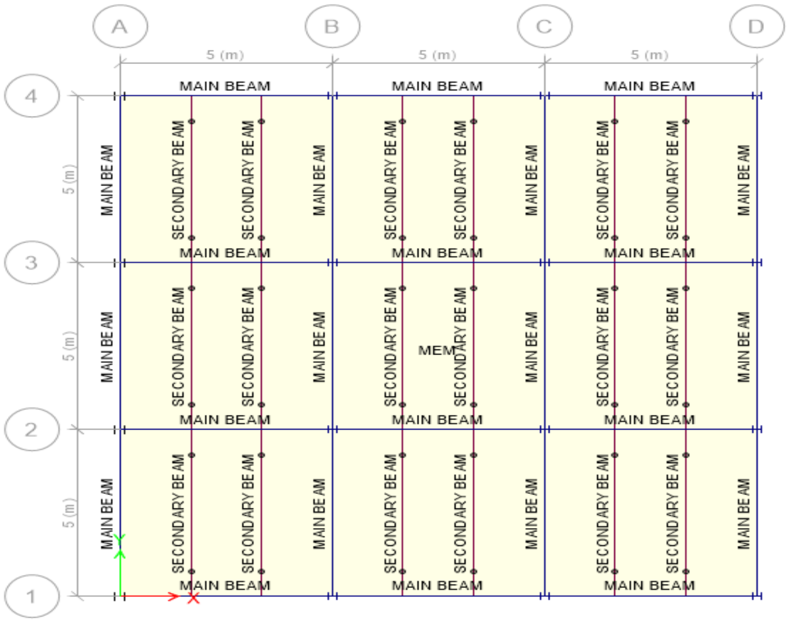
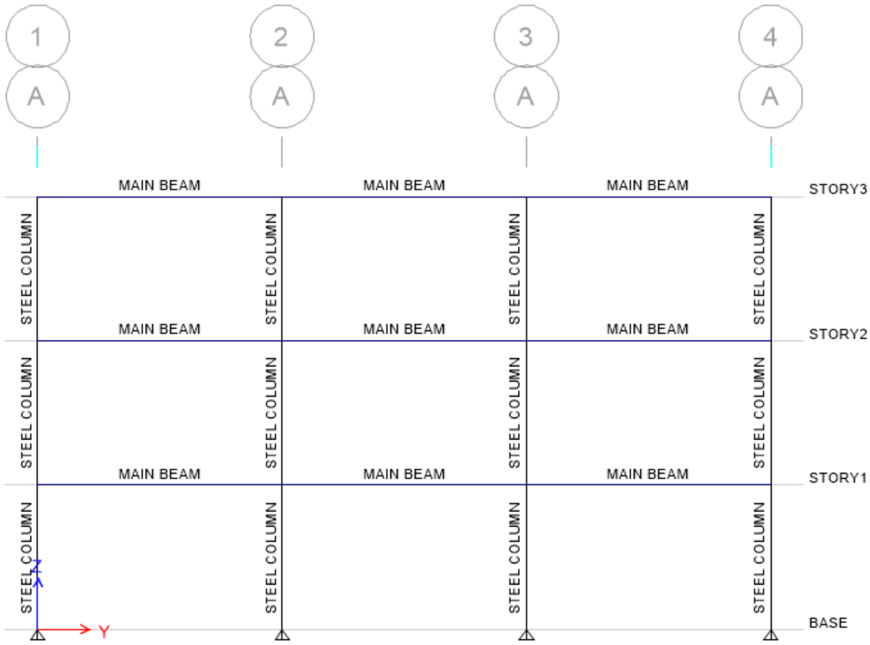


Figure 1 Steel stress-strain diagram definition in Etabs.



(a) Plan view



(b) Elevation view

Figure 2 Plan and elevation view of the ordinary moment steel frame buildings without viscoelastic damping system.

5.2 Viscoelastic Damping System Selection

The main parts of the viscoelastic dampers consisted of two layers of polymers along with outer steel flanges, as shown in Figure 3. Viscoelastic dampers offer a velocity reliant on damping force and have a consistent elastic effective stiffness, K_{eff}, with a value numerically equal to 2 times the damping coefficient, C, in units of kN/m. The viscoelastic damping coefficient ranged from 5,000 to 10,000 kN-sec/m in accordance with the constructions dissipation energy guidelines [11].

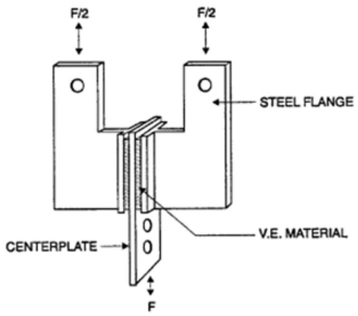


Figure 3 Viscoelastic damping element.

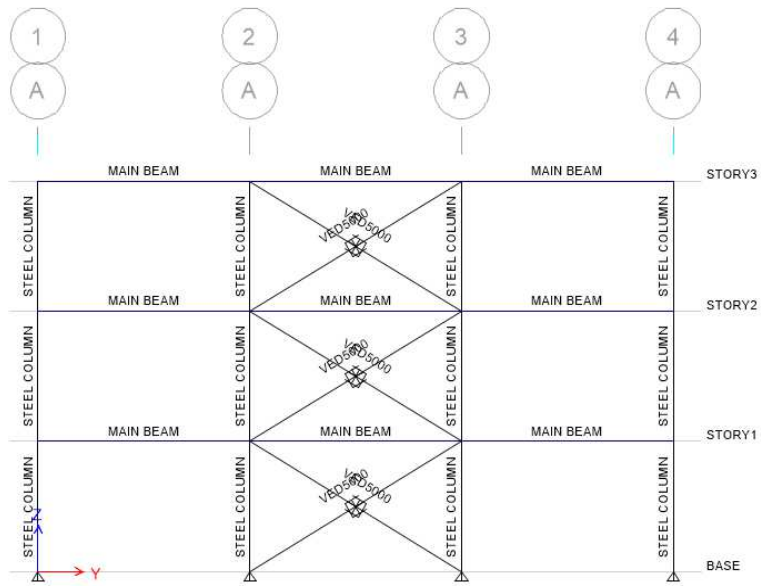
Figure 4 depicts the viscoelastic dampers used in the outer frames on every floor in the central bays, with a stiffness of 10,000 kN/m and damping coefficients of 5,000 kN-sec/m. ETABS 2013 was used to define the viscoelastic dampers, using the previous damping factor and the properties of effective stiffness [27].

5.3 Analysis Methodology

A linear time history analysis and nonlinear static pushover analysis were carried out using ETABS 2013 in the global X direction to evaluate μ, Ω, and R for each building of concern. Different earthquake records were used in the linear time history to include variability in ground motion characteristics.

Seismic weight including dead load was applied to the frames according to ASCE7-10, Section 12.7.2 and used in the linear time history analysis [28]. Linear time history analysis was used to evaluate and analyze each building of concern and to determine steel element sizes for the inelastic pushover analysis. Also, a 5% modal damping ratio was applied. According to AISC360-10, the design of a building should have the following load combinations, where D is the design dead load, L is the design live load, and E is the effect of earthquake forces as following Eqs. (7) to (10):

(a) Plan view



(b) Elevation view

Figure 4 Viscoelastic damper location in plan and elevation of ordinary moment resisting steel frame building in the outer frames.

$$U = 1.4D \tag{7}$$

$$U = 1.2D + 1.6L \tag{8}$$

$$U = 1.2D + 1.0E + 1.0L \tag{9}$$

$$U = 0.9D + 1.0E \tag{10}$$

The nonlinear static pushover analysis used the steel element sizes that were obtained from the design load combination along with the linear time history analysis. Moreover, the properties of the plastic hinges were applied along with FEMA 356 at the beam and column beginnings and ends [28].

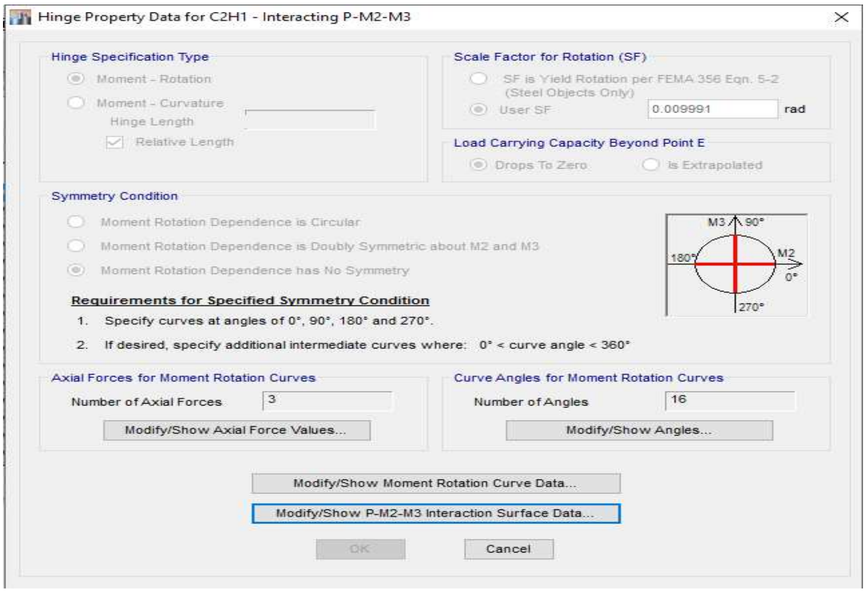
5.4 Nonlinear Pushover Analysis

The nonlinear pushover analysis was carried out under constant gravity loads and monotonically increasing horizontal loads and used to obtain the pushover curve to provide the global load deformation curve until failure is reached in the given structure. Moreover, the nonlinear static analysis was used to obtain the single force-displacement curve, also known as the pushover curve. This is known to be an incremental and iterative solution of static equilibrium equations.

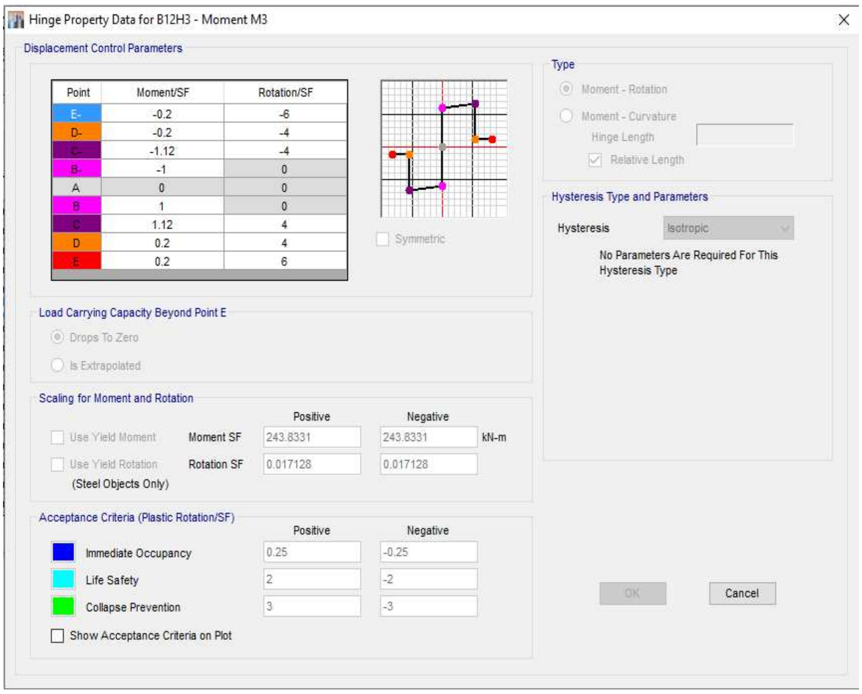
Plastic rotation occurs when material nonlinearity is assigned to a discrete hinge during the analysis of frame objects [29]. During the pushover analysis, many factors are available, including P-delta effect, staged construction and link assignment. ETABS was used to model the fiber plastic hinges along with the plastic deformation that occurs within and acts as a discrete-point hinge. ETABS was also used for the built-in default fiber hinge properties for the steel elements according to FEMA 356, which requires the section to be *I* or *H* [27]. Moreover, the nonlinear analysis should be performed after the design has been chosen. Figure 5 shows the built-in ETABS default hinge properties for different steel elements according to FEMA 356.

5.5 Seismic Records

Several seismic records have been used by engineers to investigate and explore characteristics of earthquake ground motion, studying the behavior and the response of structures to base excitation, development and evaluation of seismic codes, and for the purpose of analysis and design of earthquake resistant structures [30]. Seismic excitation records are needed for the time history analysis to incorporate site characteristics and seismic hazard. ASCE7-10 requires that a minimum of three seismic records be used in such type of analysis.



(a) Default column hinge properties.



(b) Default beam hinge properties.

Figure 5 Built-in ETABS default hinge properties for steel elements according to FEMA 356.

Modal earthquake records are shown in Figures 6 to 11. Six records were used in the time history analysis to include the variability in ground motion. Time history analysis was carried out on each building of concern using seven seismic records with different characteristics to include variability in ground motion. Different motion characteristics are shown in Table 1. Additionally, the accelerograms shown in Figure 6 through 11, also applied in ETABS 2013, were considered as the ground acceleration, with $\ddot{u}_g$ in g (gravity acceleration unit) (Y-axis), and time in seconds (X-axis) [30].

Table 1 Seismic Records Characteristics used in Time History Analysis

Earth-quake	Year	Mag-nitude	Site	Epi-central Distance (km)	Compo-nent	Max. Acc. A (g)	Max. Vel. V (m/s)	A/V Ratio	Soil Con-dition
Parkfield California	1966	5.6	Cholame, Shandon No. 5	5	N85W	0.434	0.255	1.7	Rock
San Fernando California	1971	6.4	Pacomia Dam	4	S74W	1.075	0.577	1.86	Rock
Nahanni N. W. T., Canada	1985	6.9	Site 1, Iverson	7.5	LONG	1.101	0.462	2.38	Rock
San Fernando California	1971	6.4	234 Figueroa St., LA	41	N37E	0.199	0.167	1.19	Stiff Soil
Imperial Valley California	1940	6.6	El Centro	8	S00E	0.348	0.334	1.04	Stiff Soil
Near E. Coast of Honshu, Japan	1968	7.9	Muroran Harbor	290	N00E	0.226	0.334	0.68	Stiff Soil

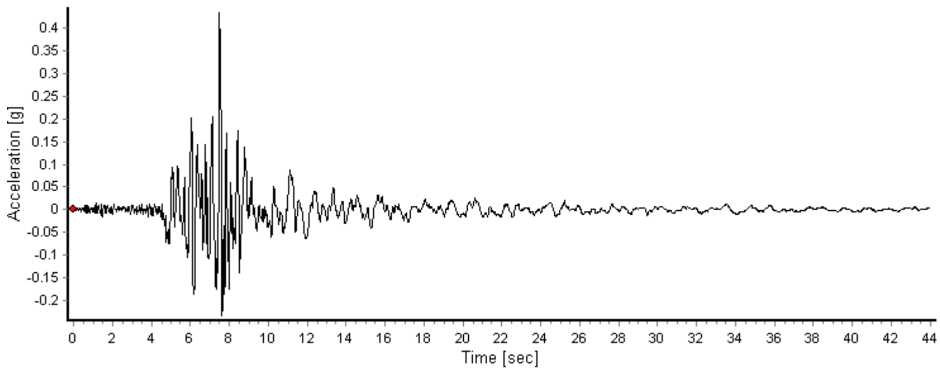


Figure 6 Parkfield California N85W earthquake accelerogram (June 27, 1966).

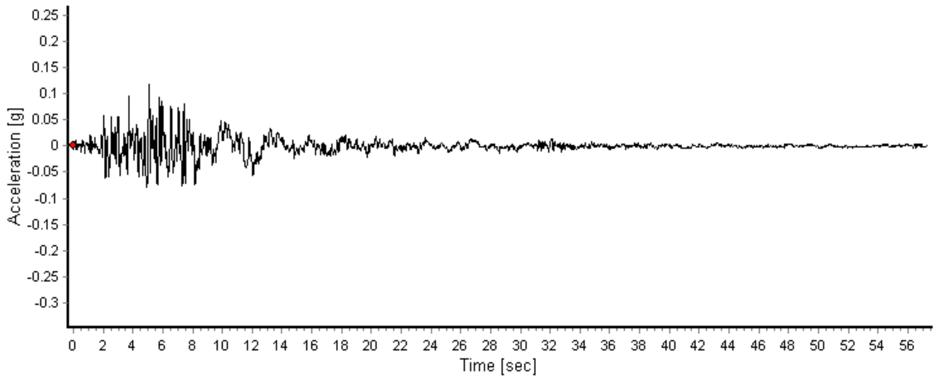


Figure 7 San Fernando California S74W earthquake accelerogram (February 9, 1971).

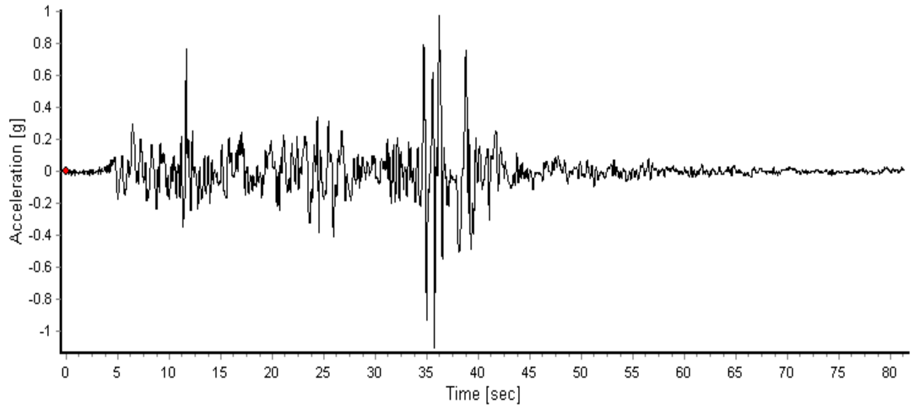


Figure 8 Canada earthquake accelerogram (December 23, 1985).

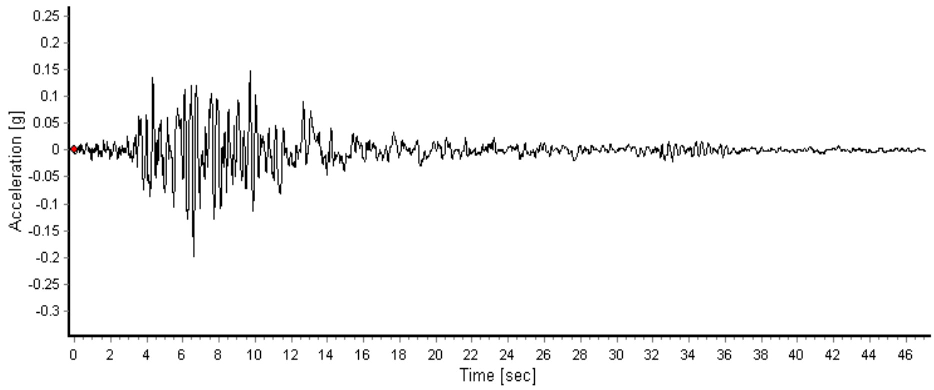


Figure 9 San Fernando California N37E earthquake accelerogram (February 9, 1971).

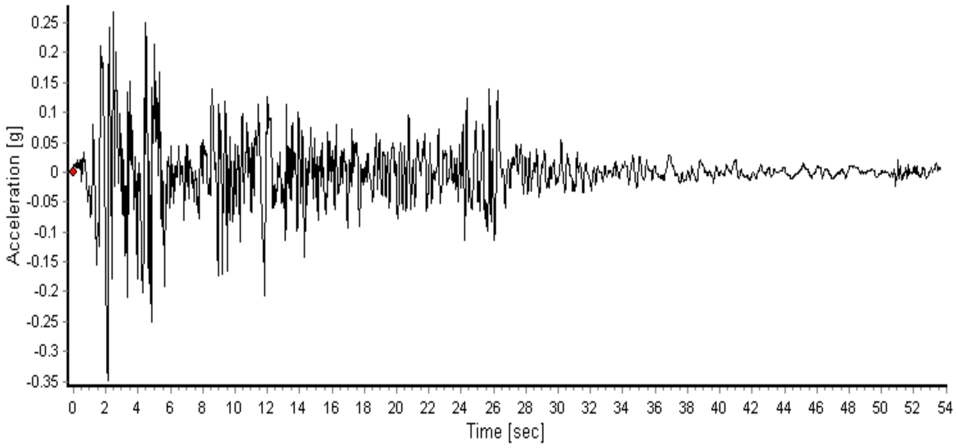


Figure 10 El Centro (California) N-S component earthquake accelerogram (May 18, 1940).

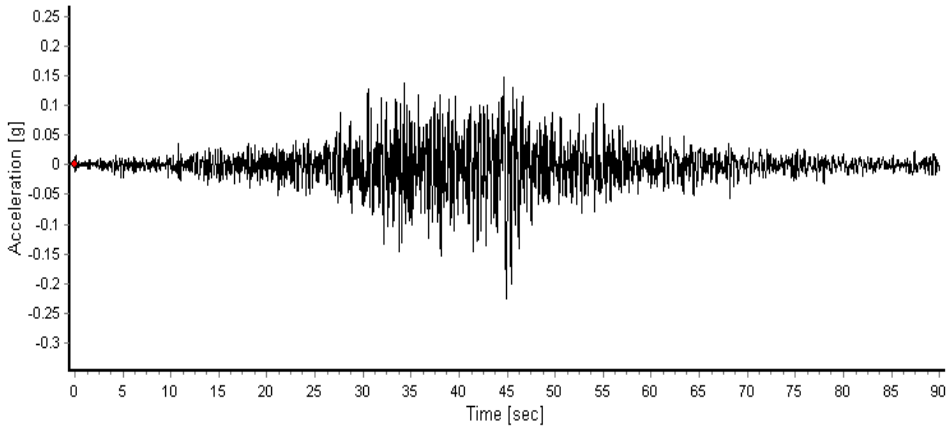


Figure 11 Near E. Coast of Honshu Japan N00E earthquake accelerogram (May 16, 1968).

6 Results and Discussions

6.1 Frame Section Sizes Summary

A linear time history analysis was performed using various seismic records to determine the steel frame section sizes (columns, main beams, secondary beams) so that these sizes could be used in the nonlinear pushover analysis. Tables 2 and 3 summarize steel section sizes for each case of concern in this study.

Table 2 Steel frame elements sizes based on number of stories.

Number of Stories	Column	Main Beam	Secondary Beam
3	HEB200	HEB200	IPE200
6	HEB260	HEB260	IPE200
9	HEB280	HEB280	IPE200
12	HEB340	HEB340	IPE200
15	HEB400	HEB400	IPE200

Table 3 Steel frame elements sizes based on span length for 3-story building.

Span Length	Column	Main Beam	Secondary Beam
5	HEB200	HEB200	IPE200
7	HEB240	HEB240	IPE240
9	HEB340	HEB340	IPE300

6.2 Static Pushover Curve

The static pushover analysis utilizes the static pushover curve, which is defined as a single force-displacement curve to evaluate the overall ductility factor of the system. It consists of an (*X*-axis), called 'monitored top roof displacement', and an (*Y*-axis) called 'base shear force' in ETABS 2013. As mentioned before, obtaining the yield and ultimate force and displacement from the pushover curve can be useful to evaluate the overall ductility factor of the system.

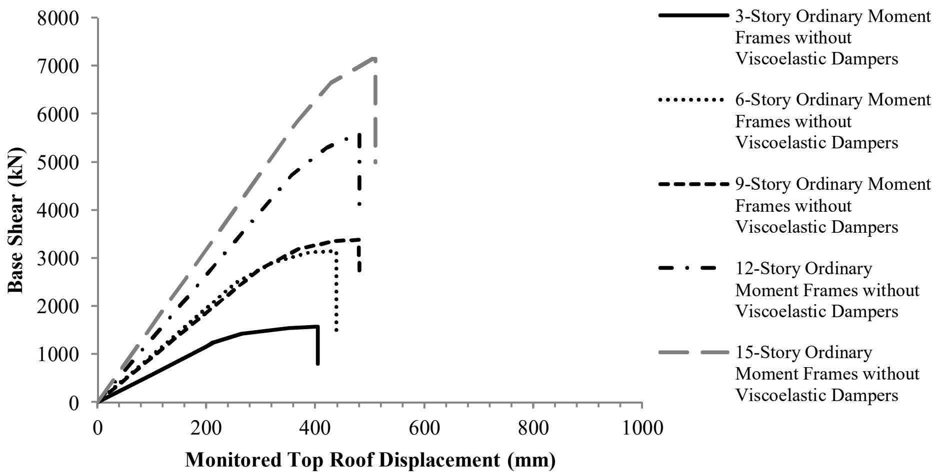


Figure 12 Static pushover curve of ordinary moment frame buildings without viscoelastic dampers for different number of stories [31].

As mentioned before, ETABS 2013 gives the static pushover curves of buildings by performing a static pushover analysis. Moreover, by reaching the first critical yield displacement point at the curve, the first plastic hinge will be formed in the structure of concern.

Figure 12 shows the static pushover curves for the ordinary moment steel frame buildings with three, six, nine, twelve, and fifteen stories without viscoelastic dampers, while Figure 13 shows the static pushover curves for the ordinary moment steel frame buildings with three, six, nine, twelve, and fifteen stories with viscoelastic dampers.

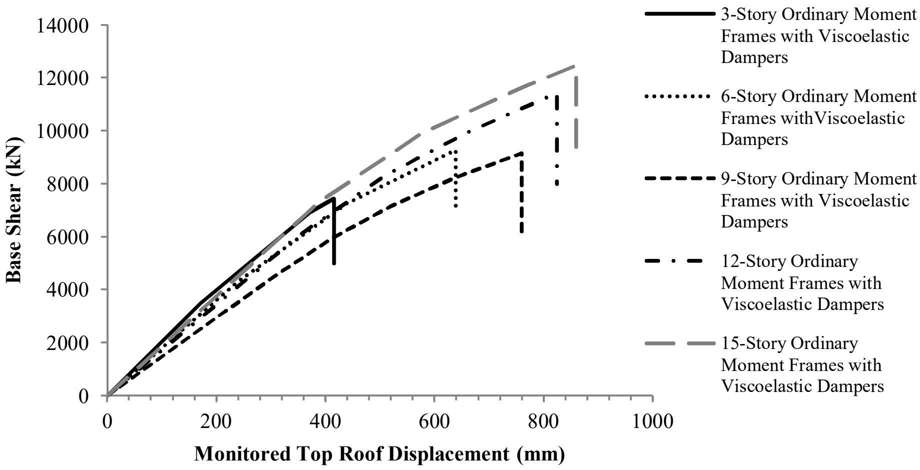


Figure 13 Static pushover curve of ordinary moment frame buildings with viscoelastic dampers for different number of stories [31].

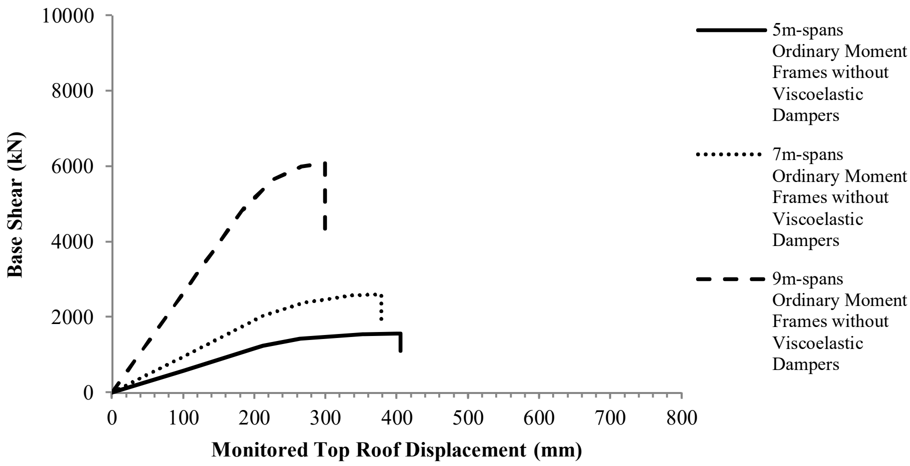


Figure 14 Static pushover curve of ordinary moment frame building without viscoelastic dampers for different span lengths [31].

Figure 14 shows the static pushover curves for the ordinary moment steel frame buildings with a five-, seven-, and nine-meter span length without viscoelastic dampers, while Figure 15 shows the static pushover curves for the ordinary moment steel frame buildings with a five-, seven-, and nine-meter span length with viscoelastic dampers.

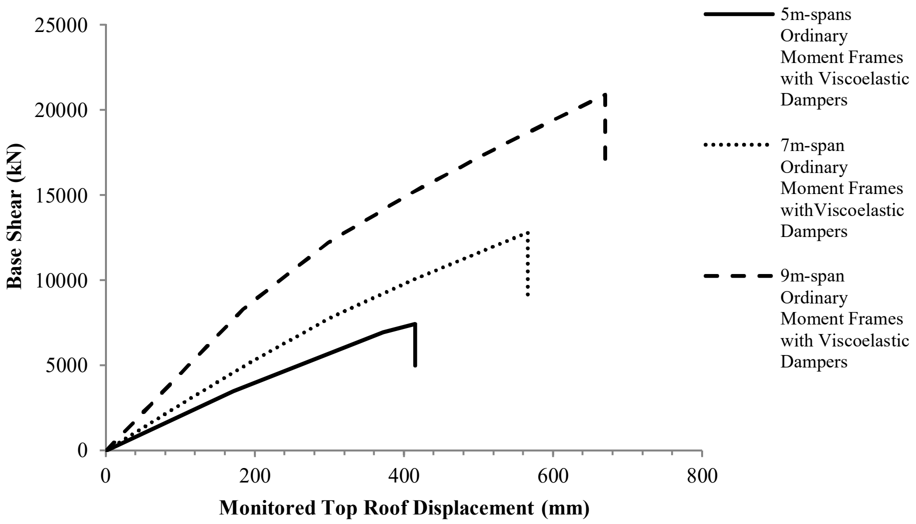


Figure 15 Static pushover curve of ordinary moment frame building with viscoelastic dampers for different span lengths [31].

6.3 Overall Ductility Factor

The overall system ductility factor, μ, can be easily characterized as the ultimate displacement divided by the corresponding displacement of the top roof when yield occurs (Δ_u/Δ_y). These two parameters can be found from the static pushover curve. The ductility factor is called the deflection amplification factor according to ASCE7-10 [28]. It is important to mention that yield displacement is measured and determined based on the formation of the first plastic hinge in the structure.

Table 4 shows the overall ductility factor for the ordinary moment steel frame buildings with three, six, nine, twelve, and fifteen stories without viscoelastic damping system as the first case. Moreover, Figure 16 shows that the overall ductility factor decreased as the number of stories increased. This result can be related to the increase of the axial compressive force on the columns by the increasing number of stories, which has the effect of decreasing the overall ductility factor.

Table 4 Overall ductility factor of ordinary moment steel frame buildings without viscoelastic dampers for different number of stories [31].

No. of stories	Yield displacement Δ_y (mm)	Ultimate displacement Δ_u (mm)	Overall ductility factor μ
3	212	404.64	1.91
6	252.5	438.50	1.74
9	296.1	479.60	1.62
12	356	485.00	1.36
15	427	510.00	1.19

Table 5 shows the overall ductility factor for the ordinary moment steel frame buildings with three, six, nine, twelve, and fifteen stories with viscoelastic damping system as the second case. Moreover, Figure 16 shows that the overall ductility factor decreased as the number of stories increased.

Table 5 Overall Ductility factor of ordinary moment steel frame buildings with viscoelastic dampers for different number of stories [31].

No. of Stories	Yield displacement Δy (mm)	Ultimate displacement Δu (mm)	Overall ductility factor μ
3	155.20	415.10	2.67
6	242.88	637.90	2.63
9	302.82	759.00	2.51
12	340.58	824.10	2.42
15	377.91	858.50	2.27

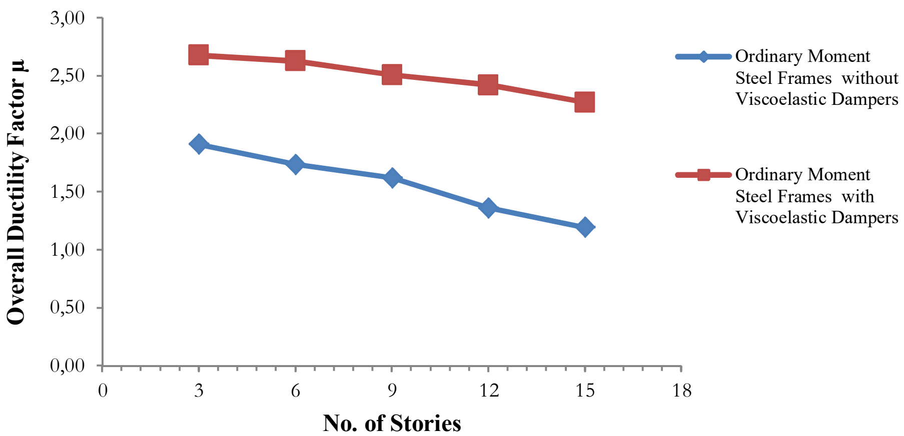


Figure 16 Overall ductility factor of ordinary moment steel frame buildings with and without viscoelastic dampers for different number of stories [31].

The effect of adding viscoelastic dampers to the system can be observed in Figure 16 by comparing the overall ductility factors of the ordinary moment steel frame buildings with three, six, nine, twelve, and fifteen stories with and without viscoelastic dampers. The results show that higher overall ductility factors were obtained when the viscoelastic damping system was added. This result can be related to an increase of the building's stiffness by adding the viscoelastic damping system, which has the effect of increasing the overall ductility factor.

Table 6 shows the overall ductility factor for the ordinary moment steel frame buildings with a five-, seven-, and nine-meter span length without viscoelastic damping system as the first case. Moreover, Figure 17 shows that the overall ductility factor decreased as the span length increased. This result can be related to the increase of axial compressive force on the columns caused by the increased bay length, which has the effect of decreasing the overall ductility factor.

Table 6 Overall ductility factor of ordinary moment steel frame buildings without viscoelastic dampers for different span lengths [31].

Span length (m)	Yield displacement Δy (mm)	Ultimate displacement Δu (mm)	Overall ductility factor μ
5	212	404.64	1.91
7	210.3	380.60	1.81
9	182.5	299.30	1.64

Table 7 shows the overall ductility factor for the ordinary moment steel frame buildings with a five-, seven-, and nine-meter span length with viscoelastic damping system as the second case. Moreover, Figure 17 shows that the overall ductility factor decreased as the span length increased.

Table 7 Overall ductility factor of ordinary moment steel frame buildings with viscoelastic dampers for different span lengths [31].

Span length (m)	Yield displacement Δy (mm)	Ultimate displacement Δu (mm)	Overall ductility factor μ
5	155.2	415.10	2.67
7	247.61	565.80	2.29
9	303.73	669.30	2.20

The effect of adding viscoelastic dampers to the system can be observed in Figure 17 by comparing the overall ductility factors of the three-story ordinary steel moment frame building with a five-, seven-, and nine-meter span length with and without viscoelastic dampers. The results show that higher overall ductility was obtained when the viscoelastic damping system was added. This result can be related to the increase in building stiffness by adding the viscoelastic damping system, which has the effect of increasing the overall ductility factor.

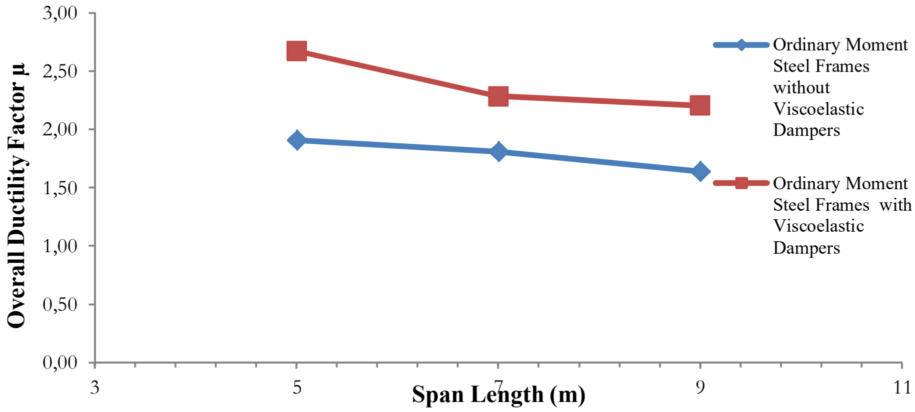


Figure 17 Overall ductility factor of ordinary moment steel frame buildings with and without viscoelastic dampers for different span lengths [31].

6.4 Overstrength Factor

The overstrength factor, Ω, is the maximum base shear divided by the design or yield base shear when yield occurs (V_m/V_y). These two parameters can be found from the static pushover curve. It is important to mention that the yield base shear is measured and determined based on the formation of the first plastic hinge in the structure.

Table 8 Ω-factor of ordinary moment steel frame buildings without viscoelastic dampers for different number of stories.

No. of stories	Yield base shear V_y (kN)	Max base shear V_m (kN)	Overstrength factor Ω
3	1237.10	1570.10	1.27
6	2481.57	3150.00	1.27
9	2759.99	3382.24	1.23
12	4726.50	5596.25	1.18
15	6641.71	7152.63	1.08

Table 8 shows the Ω-factor for the ordinary moment steel frame buildings with with three, six, nine, twelve, and fifteen stories without viscoelastic damping system as the first case. Moreover, Figure 18 shows that the Ω-factor decreased as the number of stories increased. This result can be related to the increase in axial compressive force on the columns caused by the increasing number of stories, which has the effect of decreasing the **Ω**-factor.

Table 9 shows the **Ω**-factor for the ordinary moment steel frame buildings with three, six, nine, twelve, and fifteen stories with viscoelastic damping system as the second case. Moreover, Figure 18 shows that the **Ω**-factor decreased as the number of stories increased.

Table 9 Ω-factor of ordinary moment steel frame buildings with viscoelastic dampers for different number of stories.

No. of stories	Yield base shear V_y (kN)	Max base shear V_m (kN)	Overstrength factor Ω
3	3494.15	7431.71	2.13
6	5086.20	10261.21	2.02
9	5920.09	10769.97	1.82
12	8324.90	13919.51	1.67
15	9936.37	15600.00	1.57

The effect of adding viscoelastic dampers to the system can be observed in Figure 18 by comparing the **Ω**-factor of ordinary moment steel frame buildings with three, six, nine, twelve, and fifteen stories with and without viscoelastic dampers. The results show that a higher **Ω**-factor was obtained when the viscoelastic damping system was added. This result can be related to the increase in building stiffness caused by adding the viscoelastic damping system, which has the effect of increasing the **Ω**-factor.

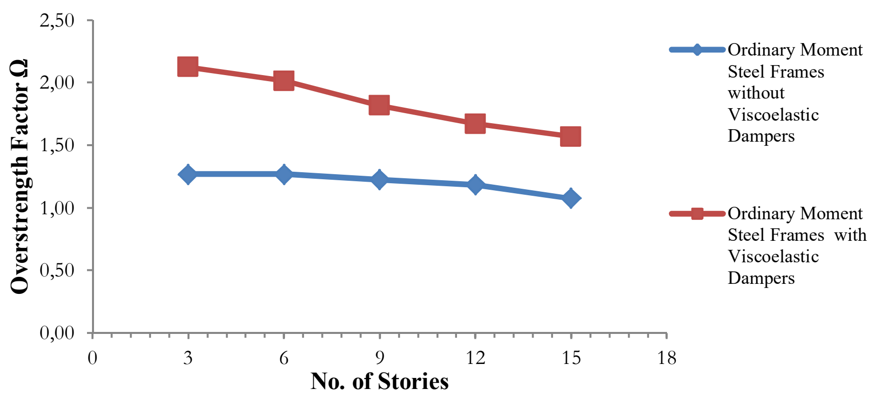


Figure 18 Ω-factor of ordinary moment steel frame buildings with and without viscoelastic dampers for different number of stories.

Table 10 shows the Ω-factor for the three-story ordinary moment steel frame buildings with a five-, seven-, and nine-meter span length without viscoelastic damping system as the first case. Moreover, Figure 19 shows that the Ω-factor decreased as the span length increased. This result can be related to the increase in axial compressive force on the columns caused by the increasing bay length, which has the effect of decreasing the Ω-factor.

Table 10 Ω-factor of ordinary moment steel frame buildings without viscoelastic dampers for different span lengths.

Span Length (m)	Yield base shear V_y (kN)	Max base shear V_m (kN)	Overstrength factor Ω
5	1237.10	1570.10	1.27
7	2118.89	2597.58	1.23
9	5093.97	6089.25	1.20

Table 11 shows the Ω-factor for the three-story ordinary moment steel frame buildings with a five-, seven-, and nine-meter span length with viscoelastic damping system as the second case. Moreover, Figure 19 shows that the Ω-factor decreased as the span length increased.

Table 11 Ω-factor of ordinary moment steel frame buildings with viscoelastic dampers for different span lengths.

Span length	Yield base shear V_y (kN)	Max base shear V_m (kN)	Overstrength factor Ω
5	3494.15	7431.71	2.13
7	7834.22	13714.60	1.75
9	15130.44	25473.74	1.68

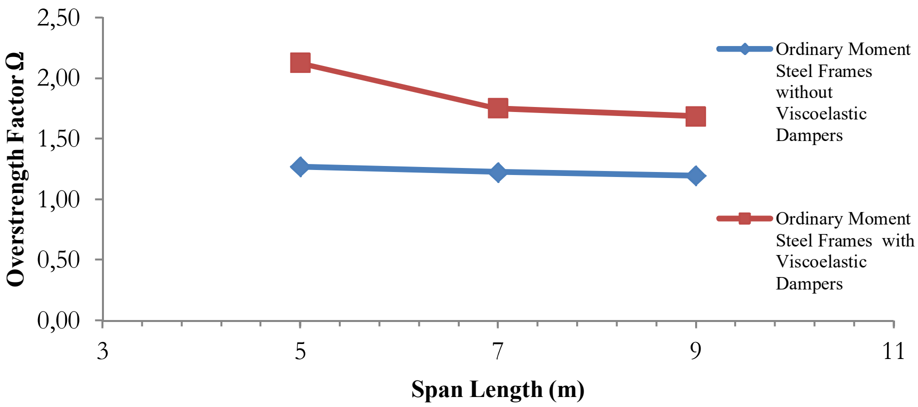


Figure 19 Ω-factor of ordinary moment steel frame buildings with and without viscoelastic dampers for different span lengths.

The effect of adding viscoelastic dampers to the system can be observed in Figure 19 by comparing the Ω-factors for the three-story ordinary moment steel frame buildings with a five-, seven-, and nine-meter span length with and without viscoelastic dampers. The results show that higher Ω-factors obtained when the viscoelastic damping system was added. This result can be related to the increase in building stiffness by adding the viscoelastic damping system, which has the effect of increasing the Ω-factor.

6.5 Response Modification Factor

The response modification factor, R, can be calculated by multiplying the ductility reduction factor and the overstrength factor.

Table 12 shows R-factor for the ordinary moment steel frame buildings with three, six, nine, twelve, and fifteen stories without viscoelastic damping system as the first case. Moreover, Figure 20 shows that the R-factor decreased as the number of stories increased. This result can be related to the decrease in R_μ and Ω caused by the increasing number of stories, which has the direct effect of decreasing the R-factor.

Table 12 R-factor of ordinary moment steel frame buildings without viscoelastic dampers for different number of stories.

No. of stories	T_n (sec)	μ	R_μ	Ω	R
3	1.032	1.91	1.91	1.27	2.42
6	1.150	1.74	1.74	1.27	2.20
9	1.300	1.62	1.62	1.23	1.98
12	1.441	1.36	1.36	1.18	1.61
15	1.489	1.19	1.19	1.08	1.29

Table 13 shows the R-factor for the ordinary moment steel frame buildings with three, six, nine, twelve, and fifteen stories with viscoelastic damping system as the second case. Moreover, Figure 20 shows that the R-factor decreased as the number of stories increased.

Table 13 R-factor of ordinary moment steel frame buildings with viscoelastic dampers for different number of stories.

No. of stories	T_n (sec)	μ	R_μ	Ω	R
3	0.545	2.67	2.67	2.13	5.69
6	0.839	2.63	2.63	2.02	5.30
9	1.1	2.51	2.51	1.82	4.56
12	1.257	2.42	2.42	1.67	4.05
15	1.363	2.27	2.27	1.57	3.57

The effect of adding viscoelastic dampers to the system can be observed in Figure 20 by comparing the R-factors of the three-, six-, nine-, twelve-, and fifteen-story ordinary moment steel frames with and without viscoelastic dampers. The results show that a higher R-factor was obtained when the viscoelastic damping system was added. This result can be related to the increase in R_μ and Ω by adding the viscoelastic damping system, which has the effect of increasing the R-factor.

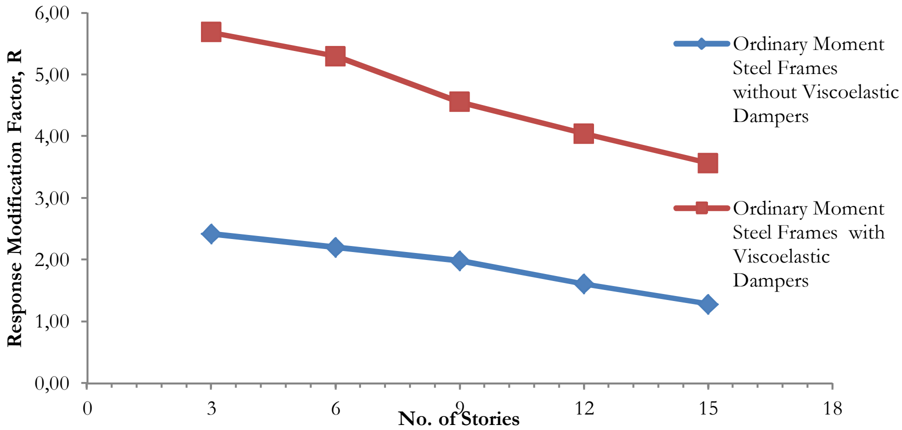


Figure 20 R-factor of ordinary moment steel frame buildings with and without viscoelastic dampers for different number of stories.

Table 14 shows the R-factor for the three-story ordinary moment steel frame buildings with a five-, seven-, and nine-meter span length without viscoelastic damping system as the first case. Moreover, Figure 21 shows that the R-factor decreased as the span length increased. This result can be related to the decrease in R_μ and Ω caused by the increasing span length, which has the direct effect of decreasing the R-factor.

Table 14 R-factor of ordinary moment steel frame buildings without viscoelastic dampers for different span lengths.

Span length (m)	T_n (sec)	μ	$R\mu$	Ω	R
5	1.032	1.91	1.91	1.27	2.42
7	1.121	1.81	1.81	1.23	2.22
9	1.251	1.64	1.64	1.20	1.96

Table 15 shows R-factor for the three-story ordinary moment steel frame buildings with a five-, seven-, and nine-meter span length with viscoelastic damping system as the second case. Moreover, Figure 21 shows that the *R*-factor decreased as the span length increased.

Table 15 *R*-factor of ordinary moment steel frame buildings with viscoelastic dampers for different span lengths.

Span length (m)	T_n (sec)	μ	$R\mu$	Ω	*R*
5	0.545	2.67	2.67	2.13	5.69
7	0.668	2.29	2.29	1.75	4.00
9	0.680	2.20	2.20	1.68	3.71

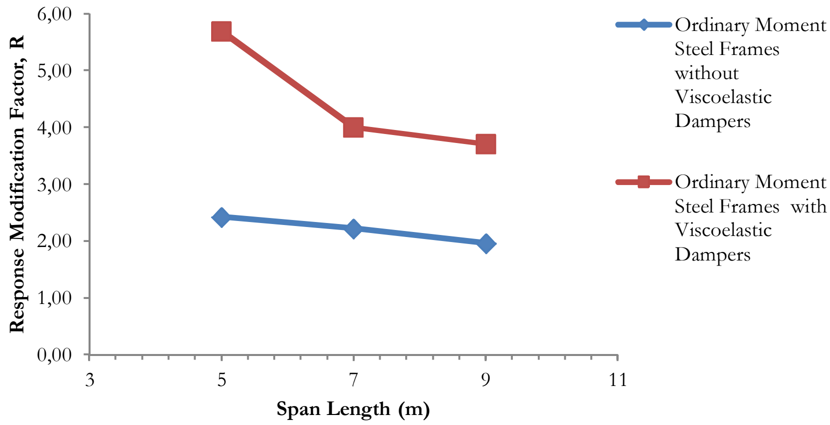


Figure 21 *R*-factor of ordinary moment steel frame buildings with and without viscoelastic dampers for different span lengths.

The effect of adding viscoelastic dampers to the system can be observed in Figure 21 by comparing the *R*-factors of the three-story ordinary moment steel frame buildings with a five-, seven-, and nine-meter span length with and without viscoelastic dampers. The results show that higher *R*-factors were obtained when the viscoelastic damping system was added. This result can be related to the increase in R_μ and Ω by adding the viscoelastic damping system, which has the effect of increasing the *R*-factor.

6.6 Elastic Building Displacement Comparison

A dynamic linear time history analysis was used to evaluate the elastic building displacement, Δ_e, measured at the rooftop for all buildings. Table 16 presents the values of elastic displacement for the 3-, 6-, 9-, 12-, and 15-story buildings with and without viscoelastic dampers. The same values are plotted in Figure 22. Increasing the number of stories increases the elastic displacements for ordinary moment steel frames and adding viscoelastic dampers to ordinary moment steel frames decreased the elastic displacement for all buildings significantly because the viscoelastic dampers increase the building's stiffness, which decreases lateral elastic movement.

Table 16 Elastic displacement of ordinary moment steel frame buildings with and without viscoelastic dampers for different number of stories.

No. of stories	Δ_e for ordinary moment frames without viscoelastic dampers (mm)	Δ_e for ordinary moment frames with viscoelastic dampers (mm)
3	128	11
6	138.3	23.1
9	155.6	45.8
12	167.1	60.7
15	186.9	78.6

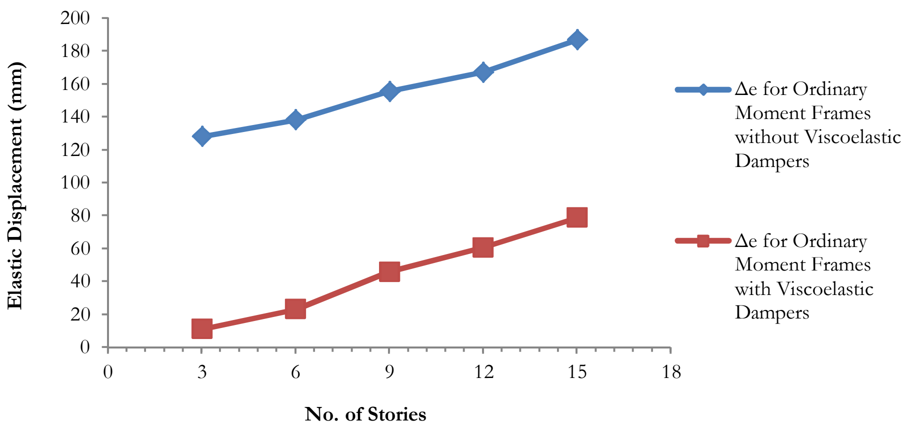


Figure 22 Comparison of elastic displacement of ordinary moment steel frame buildings with and without viscoelastic dampers for different number of stories.

Table 17 Elastic displacement of ordinary moment steel frame buildings with viscoelastic dampers for different span lengths.

Span length	Δ_e for ordinary moment frames without viscoelastic dampers (mm)	Δ_e for ordinary moment frames with viscoelastic dampers (mm)
5	128	11
7	101.9	14.8
9	83.9	15.3

Table 17 presents the values of elastic displacement f for the three-story buildings with a five-, seven-, and nine-meter span length with and without viscoelastic dampers. The same values are plotted in Figure 23. Increasing the number of stories increases the elastic displacement for ordinary moment steel frames and adding viscoelastic dampers to ordinary moment steel frames significantly decreased elastic displacement for all span lengths.

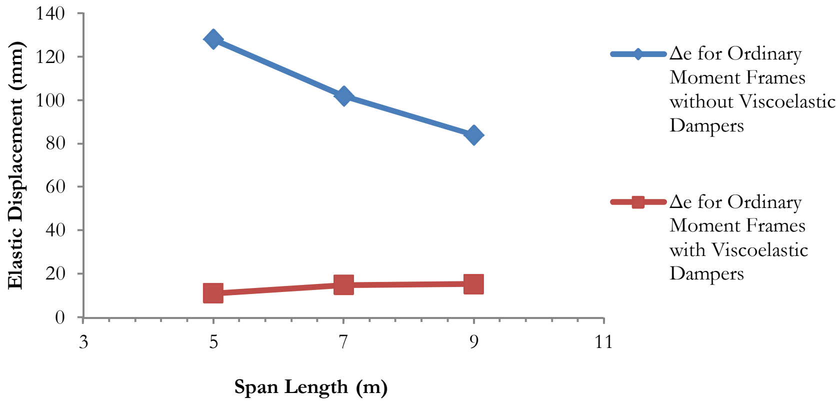


Figure 23 Comparison of elastic displacement of ordinary moment steel frame buildings with and without viscoelastic dampers for different span lengths.

7 Conclusions

In this study the seismic behavior factors μ, Ω, R of ordinary moment resisting steel frames with and without viscoelastic bracing system were investigated. The effects of the number of stories and bay length on these factors were investigated as well. It was found that μ, Ω, R decreased with an increase of the number of stories for all buildings. This result can be related to the increase in axial compression force in the columns. In addition, these factors increased when providing the viscoelastic damping system because of the increase in structural stiffness. The results also showed that elastic displacement increased by increasing the number of stories and significantly decreased when viscoelastic dampers were provided.

It was found that seismic behavior factors μ, Ω, R factors decreased as the span length increased and increased when viscoelastic dampers were provided. The results showed that elastic displacement decreased by increasing the span length and decreased by providing viscoelastic dampers.

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